

# DESIGN AND FULL SCALE TRIAL OF A LARGE SPAN ARCH EXPLOSIVE STOREHOUSE

by  
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## ABSTRACT

As part of an investigation into the development of more cost effective explosives storehouses, the Australian Department of Defence have conducted a number of trials to confirm the performance of large span circular arch explosives storehouses at Woomera.

The storehouses consisted of a proprietary profiled light gauge steel arch faced with sprayed concrete. The light gauge steel shell thus acted as expendable formwork and also contributed to the reinforcement of the arch. The resulting structure was covered with earthfill in accordance with standard specifications.

In two previous trials, three 13 m span receptor arch structures successfully withstood the blast effects generated by the detonation of 75,000 kg TNT. More recently the performance of a new 23 m span arch explosives storehouse was examined. The new receptor was located at minimum side to side separation distance from one of the 13 m arch structures remaining from the earlier trials.

The receptor structure survived the blast with only limited damage. Measured blast loads on the receptor, although less than the blast load criteria adopted in the design, are considered representative of the blast load environment to be expected. Analytical response predictions were found to be broadly in agreement with the measured values. The structure has now been approved for use as an igloo by the Australian Defence Forces.

## 1. INTRODUCTION

In view of the proposed redevelopment of a large proportion of Defence explosives storage facilities, considerable attention is being paid to the development of cost effective explosives storehouses (ESH). A number of trials, in May 1990 (Ref. 1), September 1990 (Ref. 3) and October 1991 (Ref. 3), have been conducted to assist in this process. The most recent trial is the subject of this paper. In this trial the behaviour of a large span arch explosives storehouse subjected to the effects of a detonation of 75,000 kg TNT at minimum 'side to side' separation distance was examined. The principal aim of this trial was to validate the design of the new large span arch structures as a hardened receptor structure for Explosives Storehouses spaced at D3/D4 distances (Ref. 5).

The receptor explosives storehouse consisted of an earth covered concrete (shotcrete) arch lined internally with a deep rib light gauge sheeting produced by SPANTECH. A typical plan and elevation of the structure appear as Figure 1. The unusual double arch form was used to permit a reduction in the building footprint (length)<sup>1</sup>. The

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<sup>1</sup> The smaller arch at the rear can be fitted within the earth traverse generated by the major arch whilst maintaining the minimum earth cover at all points. Thus the extra 3 m internal length does not add to the total building length.

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reinforced concrete headwall and endwalls are of conventional construction - the internal SPANTECH sheeting acts both as permanent formwork, thereby speeding construction, and also as arch reinforcement. The configuration adopted permits the storage of some 380 pallets.

The arch is constructed using light gauge (1.0 mm G300 steel plate) profiled galvanised sheeting which is then covered with a layer of 32 MPa shotcrete and lightly reinforced with steel mesh. The shotcrete thickness varies from 250 mm at the centre of the arch to 350 mm at supports. The profiled steel sheeting consists of curved trays nominally 300 mm wide with 110 mm ribs which are mechanically interlocked. Once erected the steel arch is self supporting. With the aid of propping along the centre of the arch, the arch can support the selfweight of the applied shotcrete. The headwall, the intermediate wall between the two arches and the end wall are constructed monolithically with the arches and so provide substantial support to them.

In addition to ensuring that the new receptor was capable of resisting the normal design and construction loads, the structure was designed to resist the effects of a 75,000 kg detonation at D3/D4 distances. For the side-on configuration the following blast loads were adopted for the arch crown.

Peak Side on Pressure	$P_{so}$	=	303	kPa
Blast Impulse	$i_s$	=	4272	kPa.ms
Blast Duration	$t_s$	=	28	ms
Blast Wave Velocity		=	640	m/s

For points either side of the crown, the blast loads on the arch were adjusted to approximately account for,

- . the effect of slope of the earth cover on the reflected pressures experienced.
- . the decay of the pressure wave with distance from the source

Design review of the headwall and doors was based on TM5-1300 (Ref. 5) whereas the arch itself was reviewed using a linear elastic finite element package.(Ref. 6). In view of the substantial circumferential compressive stresses developed in the arch and the limited capacity of the shell in these circumstances to behave in a ductile manner, the review basis adopted was to ensure that the combined axial and flexural loads remained substantially within the ultimate load interaction diagram for the section.

## 2. TEST CONFIGURATION

The test configuration adopted for the trial is set out in Figure 2. The donor structure used in the trial was one of the existing 13 m arch ESH trialled previously. The donor consisted of a 250 mm reinforced concrete arch with a 300 mm rear wall. The headwall was of 350 mm reinforced concrete and incorporates 900 x 500 buttresses at the door. A sliding steel blast door was centrally located. Earth cover geometry was in accordance with ESTC Leaflet No. 6, Ref. 4., namely 600 mm cover at the roof with a 1:2 slope back to natural surface. The fill consisted of the soil readily available at the site - a heavy clay. Details of this structure appear in Reference 1.

Although the use of a 13 m arch as the donor meant that the trial did not exactly represent the blast environment that might be expected from a 'large span SPANTECH' ESH it was considered that the blast loads were likely to be acceptable as a basis for judging the performance of the receptor.<sup>2</sup>

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<sup>2</sup> As the internal volume of the donor SPANTECH structure is only 520 m<sup>3</sup>, the effective charge density, assuming 75000 kg HE, is 144 kg/m<sup>3</sup>. The internal volume of the new receptor structure is 1650 m<sup>3</sup> and so, for 75000 kg, the explosive loading density is very much less. The blast loads generated in this trial should therefore be more severe than those likely to be generated by a detonation in a 23 m ESH.

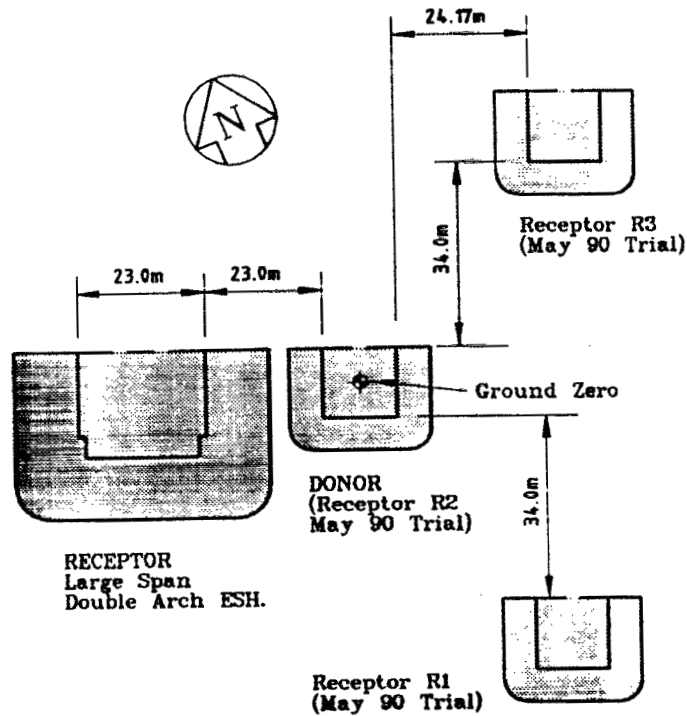


Figure 2. ESH Donor/Receptor Trial. Site Plan.

## 2.1 EXPLOSIVE CHARGE

The donor charge consisted of 120 pallets of boxed mines, arranged in a roughly semi-cylindrical configuration as shown in Figure 3. The priming charge geometry was determined to ensure a complete and instantaneous detonation, thereby eliminating the risk of 'throw outs' or unexploded mines contaminating the site.

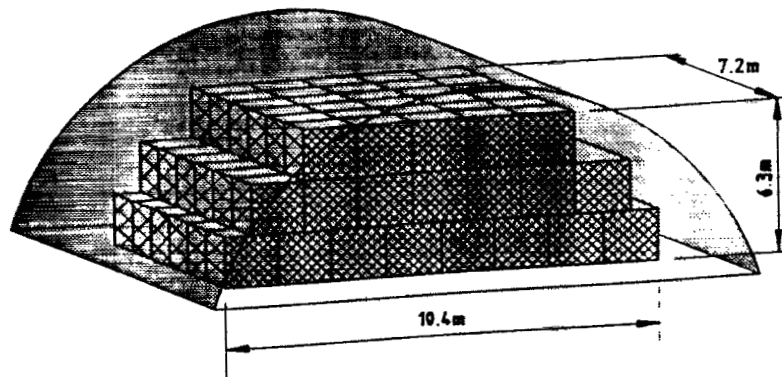


Figure 3. Explosive Stack Configuration.

## 2.2 INSTRUMENTATION

Explosives Ordnance Division (EOD), of Materials Research Laboratory (MRL) was tasked with measuring the blast overpressures on and inside the receptor structure as well as the structural accelerations and displacements of the receptor building. Additional details regarding the EOD instrumentation is provided in the EOD report (Ref. 8).

Waterways Experiment Station (WES), US Department of Army Corps of Engineers supplemented the near field instrumentation effort by locating a number of pressure transducers on the new receptor and also on the receptors remaining from earlier trials. In addition WES provided the instrumentation for the far field pressure investigation. Additional details regarding the WES instrumentation is provided in the WES preliminary report (Ref. 7).

### 2.2.1 Far Field Measurements

Gauge lines for the far field measurements were set out with respect to the nominal centre of the donor structure as displayed in the following figure. Three radial lines were used - lines at 0° (north), 90° (east) and 180° (south). (The 0° line is taken as the direction forward of and perpendicular to the original donor headwall). The nearer gauges are located at standard quantity distances and thus the distances relate to the distance from the structure walls or footing.

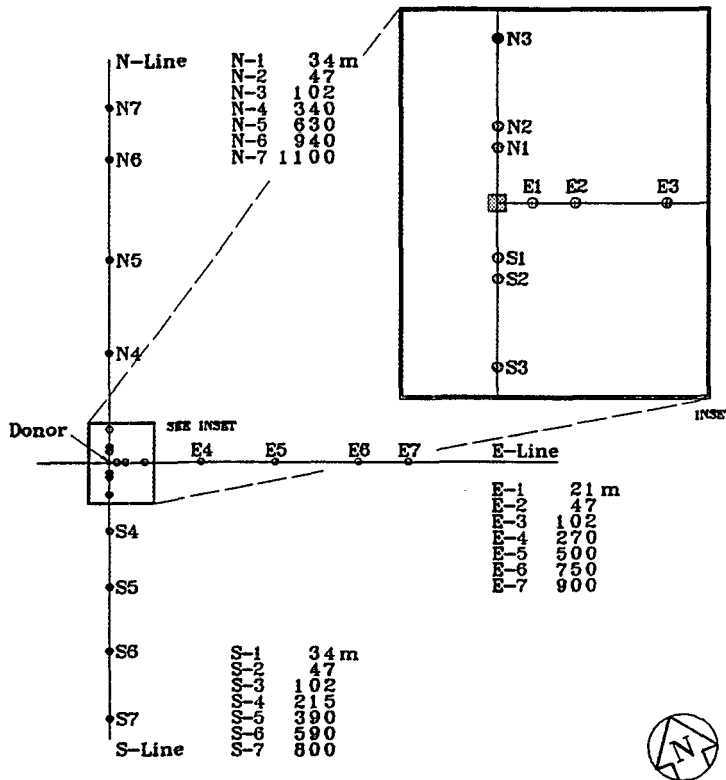


Figure 4. Layout of Far Field Pressure Gauges.

### 2.2.2 Near Field Instrumentation

In order to assess the behaviour of the receptor structure under the blast load, instrumentation was located on and inside the receptor structures to record the blast pressures applied to the structure and also the response of the structure to these loads.

A total of 13 pressure gauges - comprising 9 on the roof, 3 on the headwall and one internal were deployed by EOD as shown in Figures 5 and 6 respectively. An additional eight near field pressure gauges were deployed by WES. Four were located on and adjacent to the 13 m ESH remaining from earlier trials. The other four were located on and adjacent to the new receptor

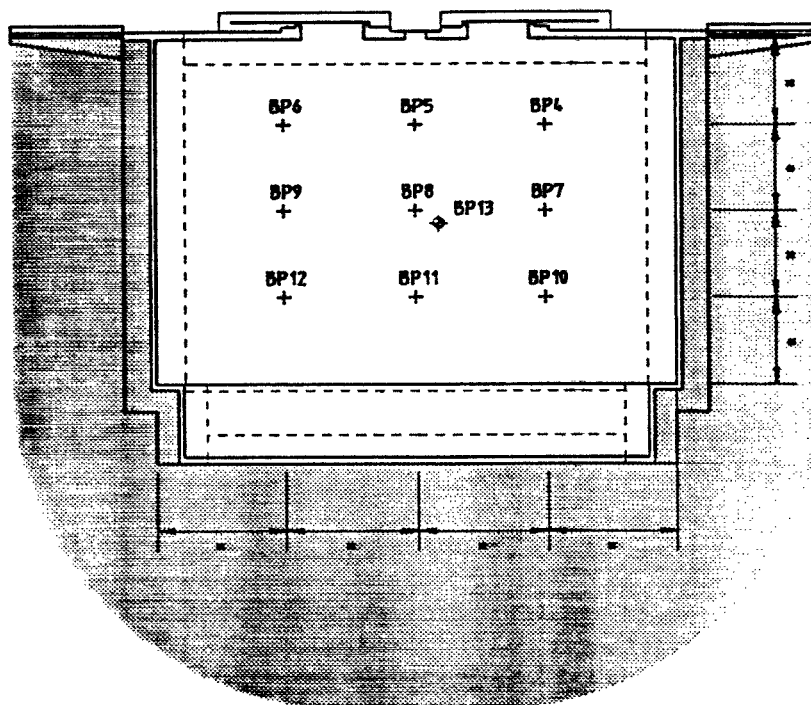


Figure 5. Plan View of Near Field Pressure Gauges on the Large Span Receptor Roof.

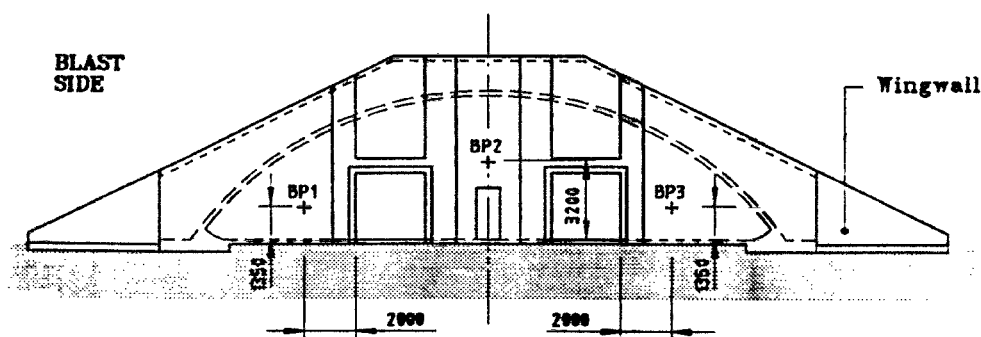


Figure 6. Layout of Pressure Gauges on the Receptor Headwalls.

In view of the difficulties in the former trials in determining the displacement response of the arch from accelerometer records, the large span Receptor was instrumented with a number of Sangamo UAC50 Linear Variable Displacement Transducer (LVDT). The transducers are an AC captive armature type with a  $\pm 50$  mm stroke and fitted with a universal joint at each end. The displacement transducers were connected to softly sprung inertial mounts in order to obtain an indication of the absolute displacement of the attachment point. The

transducers were generally arranged in pairs as shown in Figure 7, to measure vertical and horizontal displacements on the arch.

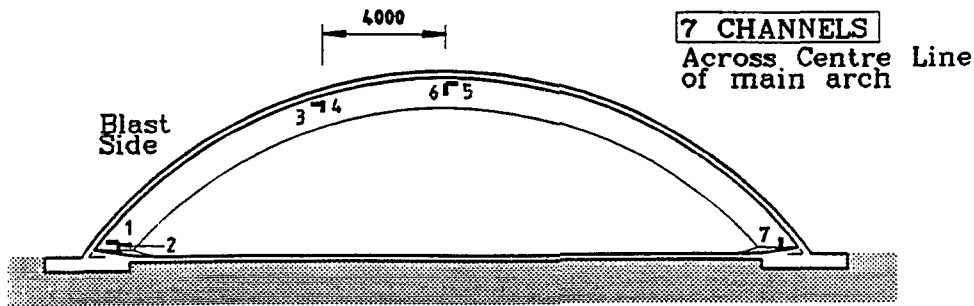


Figure 7. Location of Displacement Transducers. (Main Arch)

### 3. TEST RESULTS

The donor charge was detonated at approximately 9:30 am on 26 October 1991. Based on the instrumental results for the far field pressures it is considered that complete detonation was achieved. The detonation resulted in the complete demolition of the donor structure - all that remained was the crater (and extensive earth uplift around its perimeter). The maximum apparent depth of the crater is approximately 2.0 m with a 'diameter' varying from approximately 25 m to 30 m. Although comparable in size to those observed in the former trials the resultant crater is very much less extensive than might be expected from standard correlation expressions, eg as in CONWEP Ref. 9, for a free field surface detonation.

The large span receptor survived the detonation with only minor damage. The external face of the structure was not even blackened by soot from the fireball (notwithstanding its proximity to the donor). An examination of the headwall revealed numerous fine cracks along the support provided by the arch and headwall buttress. These cracks are considered to be consistent with elastic response rather than permanent deformation.

Of greatest concern was the behaviour of the major arch. Whilst the major arch showed no signs of significant change to its internal profile it was apparent that some significant deformations had been experienced. The SPANTECH lining was 'drummy' over most of the arch surface - indicating that the sheeting was no longer in intimate contact with the concrete. In addition for the footing closest to the blast, significant crimping of the lining was experienced at the arch-footing junction.

The most significant damage experienced was the failure of the main doors to the ESH under rebound. These were discovered lying on the ground in front of the headwall after the blast. Although the doors were subjected to substantial blast loads and suffered some permanent deformation (approximately 100 mm central), the failure was not due to inadequate design but rather due to faulty workmanship in the fixings of the rebound restraints.

### 4. INSTRUMENTAL RESULTS

Two different systems were used as the basis for time zero. EOD used a 'breakwire' wrapped around the explosive. WES used the electrical firing signal. As the WES results were triggered off the electrical firing signal, rather than the break wire trigger used by EOD a timing correction was required.



## 4.1 PRESSURE MEASUREMENTS

The WES far field radial pressures are summarised in Table 1 together with estimates for an equivalent free field hemispherical detonation. Note that the distances recorded here have been adjusted to yield distances from Ground Zero (i.e. centre of the donor structure) to the gauge point. The results obtained for peak pressure are plotted against the CONWEP estimates in Figure 8. The instrumental results appear low at short scaled distances and although they appear to converge at higher scaled distances it should be noted that they are still only 60 to 70% of the free field values.

Transducer	Distance from GZ (m)	Arrival Time (ms)	Peak Pressure (kPa)	Impulse (kPa. ms)	Positive Phase Duration (ms)
<u>Northern Line</u>					
N1	40.5	n.r. (18.3)	n.r. (1477)	n.r. (9881)	n.r. (63.7)
N2	53.5	n.r. (30.5)	n.r. (806)	n.r. (8543)	n.r. (93.2)
N3	108.5	n.r. (114)	n.r. (161)	n.r. (4467)	n.r. (100)
N4	346.5	n.r. (707)	n.r. (19.6)	n.r. (1579)	n.r. (189)
N5	636.5	n.r. (1511)	n.r. (8.63)	n.r. (875)	n.r. (230)
N6	946.5	2545 (2402)	4.00 (5.24)	475 (593)	n.a. (258)
N7	1106.5	3050 (2867)	3.24 (4.28)	375 (508)	n.a. (269)
<u>Eastern Line</u>					
E1	27.5	32.5 (9.33)	517 (3124)	4260 (7443)	n.a. (19.3)
E2	53.5	71.7 (30.5)	237 (806)	3700 (8543)	n.a. (93.2)
E3	108.5	182 (114)	88.3 (161)	2520 (4467)	n.a. (100)
E4	276.5	603 (519)	21.6 (27.5)	1500 (1953)	n.a. (176)
E5	506.5	1246 (1147)	5.95 (11.6)	867 (1093)	n.a. (214)
E6	756.5	2110 (1853)	5.22 (6.95)	565 (739)	n.a. (243)
E7	906.5	2547 (2286)	4.00 (5.54)	482 (619)	n.a. (255)
<u>Southern Line</u>					
S1	40.5	44.2 (18.3)	416 (1477)	3460 (9881)	n.a. (63.7)
S2	53.5	62.8 (30.5)	213 (806)	3260 (8543)	n.a. (93.2)
S3	108.5	178 (114)	n.r. (161)	n.r. (4467)	n.a. (100)
S4	221.5	461 (375)	24.3 (39.7)	1800 (2391)	n.a. (163)
S5	396.5	940 (843)	9.58 (16.1)	1050 (1387)	n.a. (198)
S6	596.5	1644 (1398)	7.02 (9.37)	581 (932)	n.a. (226)
S7	806.5	2240 (1997)	3.95 (6.42)	435 (695)	n.a. (247)

n.r. = no result

n.a. = not available

Table 1. Summary of Far Field Pressure Measurements

Pressure Measurements taken on the new receptor, and also on the existing small span storehouses are summarised in Table 2 for the arch roof and headwall. Results are provided for the time of arrival, peak pressure, impulse and equivalent duration. To provide a frame of reference for these values the corresponding estimates for a hemispherical detonation are included in brackets. The results reported by WES are incorporated in this table also. The results obtained in this table are plotted in the next two figures. In view of the small range of scaled distances involved a linear plot scale has been used here.

Plots of the pressure measurements and positive phase impulse data together with CONWEP estimates have been plotted in Figure 9. The comparison is not strictly appropriate as the CONWEP estimates are based on a plane hemisphere whereas most of the measured near field values are on, or near, the crests of earth mounds or on slopes either facing to or away from the blast. As expected, in view of the significant effect of the details of the donor breakup on the blast environment, the results show considerable scatter and are significantly below the CONWEP line.

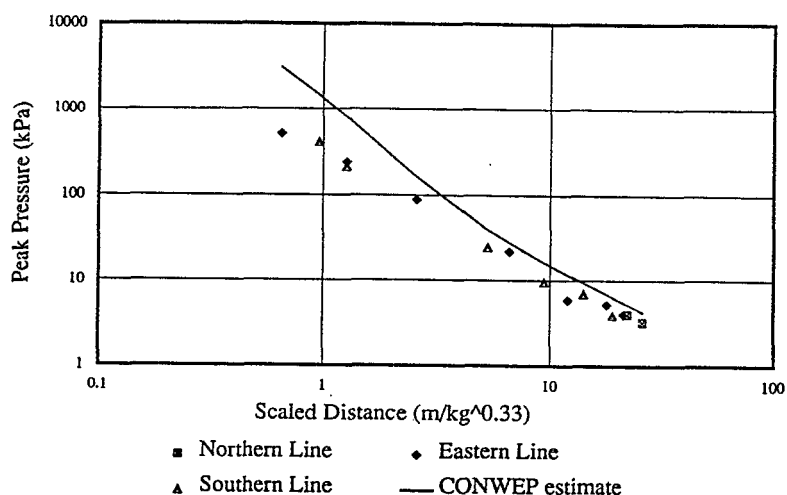


Figure 8. Comparison of Measured and Estimated Far Field Pressures.

Transducer	Distance from GZ (m)	Arrival Time (ms)	Peak Pressure (kPa)	Impulse (kPa. ms)	Equivalent Duration (ms)
<b>EOD Results</b>					
<b>Headwall<sup>3</sup></b>					
BP1	35.2	n.r. (14.3)	n.r. (1965)	n.r. (8796)	n.r. (8.9)
BP2	42.3	52.4 (19.8)	436 (1348)	4160 (9790)	19.1 (14.5)
BP3	49.4	64.2 (26.3)	352 (963)	3400 (9059)	19.3 (18.8)
<b>Arch Roof</b>					
BP4	36.3	44.2 (15.1)	>932 (1848)	n.r. (9041)	n.r. (9.8)
BP5	42.0	51.8 (19.6)	474 (1368)	2200 (9809)	9.3 (14.3)
BP6	47.7	62.0 (24.7)	240 (1041)	2350 (9264)	19.6 (17.8)
BP7	36.0	44.9 (14.8)	>930 (1879)	5490 (8973)	<11.8 (9.5)
BP8	41.8	52.6 (19.4)	573 (1382)	2220 (9821)	7.7 (14.2)
BP9	47.5	62.7 (24.5)	258 (1050)	2090 (9288)	16.2 (17.7)
BP10	36.3	45.9 (15.1)	756 (1848)	5340 (9041)	14.1 (9.8)
BP11	42.0	53.9 (19.6)	516 (1368)	2640 (9809)	10.2 (14.3)
BP12	47.7	63.8 (24.7)	251 (1041)	2080 (9264)	16.6 (17.8)
<b>Internal</b>					
BP13	41.8	n.r. (19.4)	n.r. (1382)	n.r. (9821)	n.r. (14.2)
<b>WES Results</b>					
NFP1	21.5	28.4 (6.23)	800 (4727)	6190 (7005)	15.5 (2.96)
NFP2	41.6	50.3 (19.2)	323 (1396)	2400 (9832)	14.9 (14.1)
NFP3	42.1	50.0 (19.7)	280 (1362)	4570 (9803)	32.6 (14.4)
NFP4	66.9	62.6 (46.3)	175 (484)	4770 (7014)	54.5 (28.9)
NFP5	61.6	88.9 (39.7)	255 (585)	4160 (7570)	32.6 (25.9)
NFP6	61.0	60.9 (39.0)	279 (598)	2510 (7638)	18.0 (25.5)
NFP7	57.0	n.r. (34.3)	351 (699)	>2750 (8108)	>15.7 (23.2)
NFP8	62.8	83.9 (41.2)	176 (560)	3360 (7438)	38.2 (26.6)

n.r. = no result

Table 2. Summary of Near Field Pressure Measurements

<sup>3</sup> Note that the CONWEP estimates quoted for the headwall are 'side-on' values.

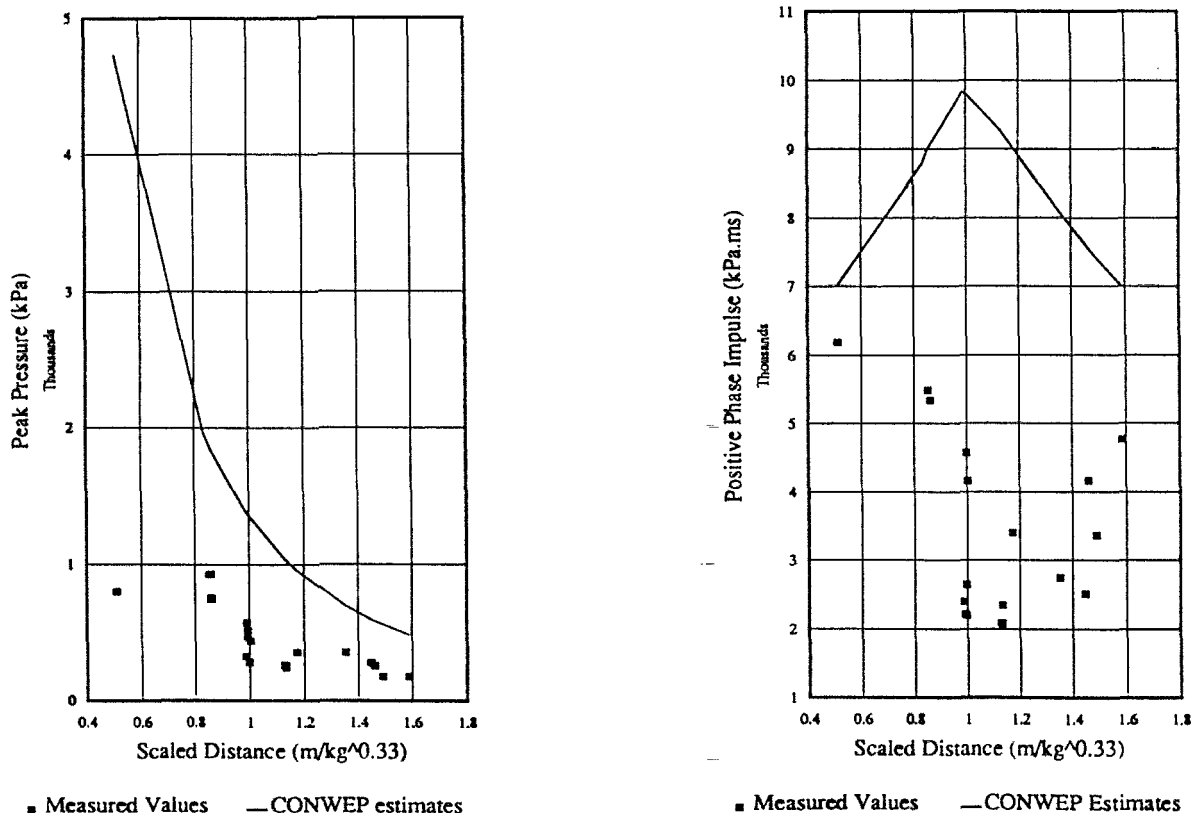


Figure 9. Comparison of Near Field Measured and Estimated Peak Pressure and Impulse Measurements.

## 4.2 STRUCTURE RESPONSE MEASUREMENTS

Instrumental measures of the ground shock and structure response were determined from the accelerometer and displacement transducer records.

### 4.2.1 Arch Response

Arch response data was obtained principally from the displacement transducers, however some supplementary accelerometer data was also obtained. Only the displacement transducer results are discussed here.

Six of the twelve displacement transducers, namely Channels 1 to 4, 6 and 8, were damaged during the blast due to the ground shock displacements over-ranging the transducers. Limit stops had been installed to prevent excessive displacement however these had only been partially successful.

Notwithstanding the damage sustained, all of the transducers functioned satisfactorily until they were over-ranged which, generally, can be detected as a relatively abrupt discontinuity in the displacement trace. This typically occurred some 150 ms to 200 ms after the detonation. It is considered that up to this point the displacement traces provide a reliable indication of the absolute motion<sup>4</sup> of the point of the structure to which the transducer was connected.

<sup>4</sup> Which consists of motion due to structure deformation and ground motion.

As an example Figure 10 compares the horizontal displacements recorded along the centre of the main arch. As noted earlier Channel 1 is located adjacent to the arch footing (blast side), Channel 6 is located at mid span, and Channel 3 is located at an intermediate point 4 m from Channel 6. The three traces can be seen to exhibit similar behaviour in the first 150 ms. Note that Channel 1 initially experiences a motion towards the blast and that the characteristics of this motion are reflected in the traces for Channels 3 and 6.

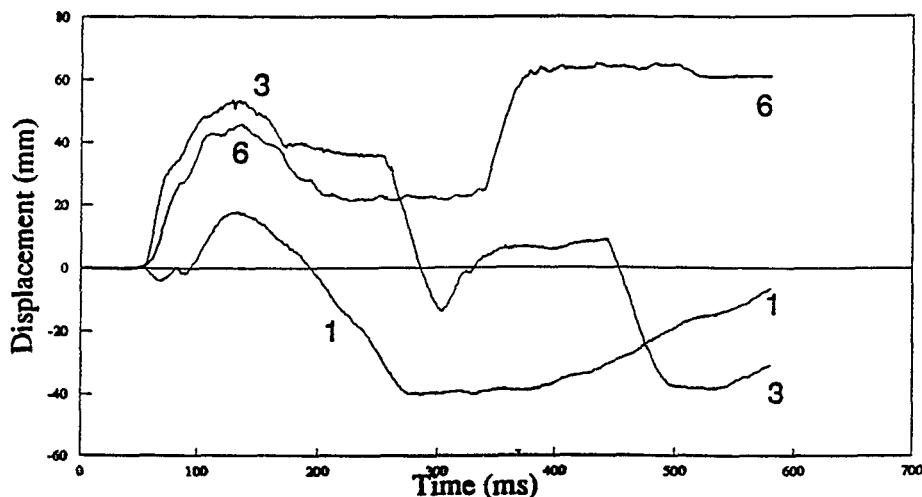


Figure 10. Displacement (horizontal) traces for Channels 1,3 and 6.  
(Note: Positive displacements indicate motion of the attachment point away from blast)

### Arch Deformations

The arch deformations are estimated by obtaining the displacements of points on the arch relative to the arch footings thus removing any rigid body rotations due to ground deformation. The following table summarizes estimates of the measured arch deformation for the three locations instrumented (three pairs of transducers). Note that, as in the figures, positive horizontal displacements correspond to motion of the attachment point away from the blast, positive vertical motions correspond to downwards motion of attachment point.

Transducer	Attitude	Significant Displacement Peaks (mm)	Time at which Maxima Occurred (ms)
Ch 3	Horizontal	+35, +43	80, 100
Ch 4	Vertical	+35	85
Ch 5	Vertical	-4.2	100
Ch 6	Horizontal	+35	106
Ch 10	Horizontal	+13	94
Ch 11	Vertical	-7	100

Table 3. Large Span Receptor - Arch Displacements.

## 5. STRUCTURAL RESPONSE PREDICTIONS

In order to assess the validity of the structural models adopted for the design of the facility, and thus assess our capacity to predict the behaviour of the arch, a number of analyses of the Receptor were performed. A linear

elastic finite element model was used to examine the behaviour of the structure to the instrumentally determined blast overpressures.

The analysis was performed using the program ALGOR/SUPERSAP. A model of the arch, is shown in Figure 11, in which the soil, modelled using 8 noded brick elements, has been partly removed to reveal the shell underneath.

Although the model developed is computationally intensive - entailing approximately 10 hours processing time on a 33 Mhz 486 microcomputer - it must be recognised that the model is nevertheless still limited in its capacity to predict the true response of the system. The results therefore should be seen only as an approximation to the real behaviour.

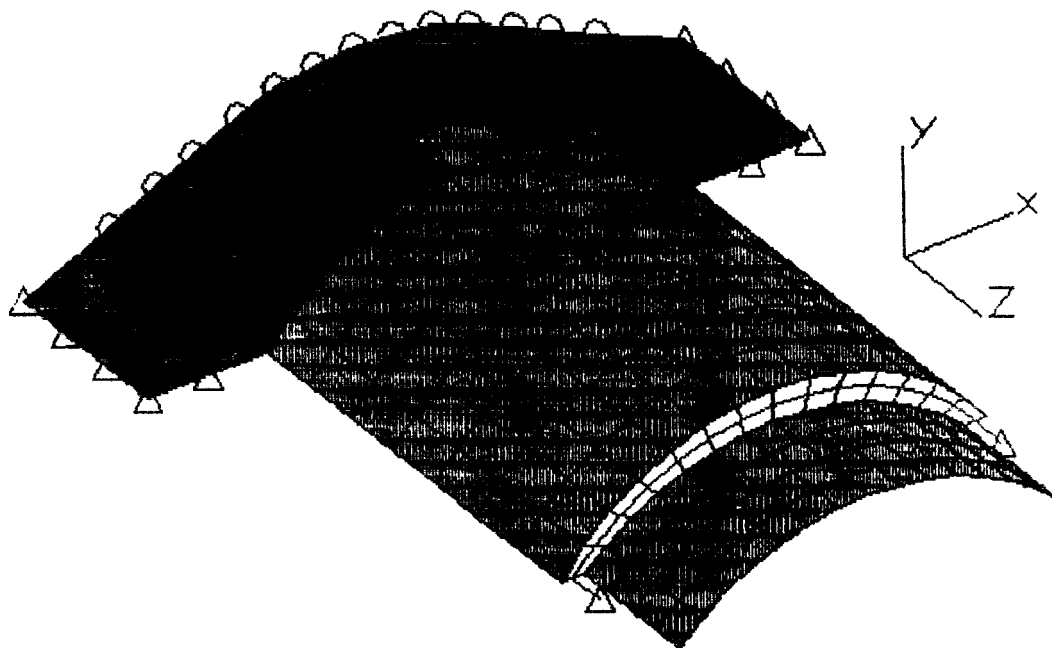


Figure 11. Structural Model Adopted for 'Side-On' Case.

## 5.1 ARCH PROPERTIES

Conventional 4 node plate elements were used to model the reinforced concrete arch. The arch footing was assumed to be elastically supported using linear elastic boundary elements. Rotational springs (based on the soil properties subsequently noted) were used to model the rotational stiffness of the soil - which, for the short duration loads involved, is considered appropriate. In addition soil springs were used to provide vertical and horizontal restraint.

Because of the deep profile SPANTECH sheeting in the soffit of the arch, the arch section will exhibit different section properties depending on the sense of flexure induced. That is, in the longitudinal direction, the effective thickness of the shell may vary from 250 mm to 140 mm depending on whether the top surface is in tension or compression. As this effect cannot be modelled with a linear elastic finite element analysis a uniform section has been used throughout. However to approximately account for this reduced flexural stiffness relative to the axial stiffness, the model has been stretched by a factor of two in the longitudinal direction. The effect can also be partially accounted for in the load capacity checks.

Load capacity checks were performed using an interaction diagram derived for typical sections. As an adequate model for the prediction of the ultimate strength behaviour of reinforced concrete slabs to a general load (axial stresses in both directions, shear stresses, bending in both directions plus twisting) does not exist, it is necessary to resort to a uniaxial environment to determine the capacity of the section. Actions in the perpendicular direction are thus assumed not to significantly affect the behaviour in the principal direction. Also, in the circumferential direction, the effective thickness is unaffected by the sense of flexure.

As the 28 day test results for the shotcrete yielded a mean strength of 39 MPa - a dynamic concrete strength of 45 MPa and a dynamic elastic modulus of 32000 MPa was used in the analysis. A value of 500 MPa was adopted for the dynamic yield strength of the reinforcement - which represents a 20% increase over the static value.

It should be noted that points falling outside the zone defined by the interaction diagram do not necessarily imply collapse of the arch. A number of supplementary issues must be considered, such as the,

- spatial extent of the overloaded region
- duration for which an element is overloaded
- low axial stresses, high flexural loads

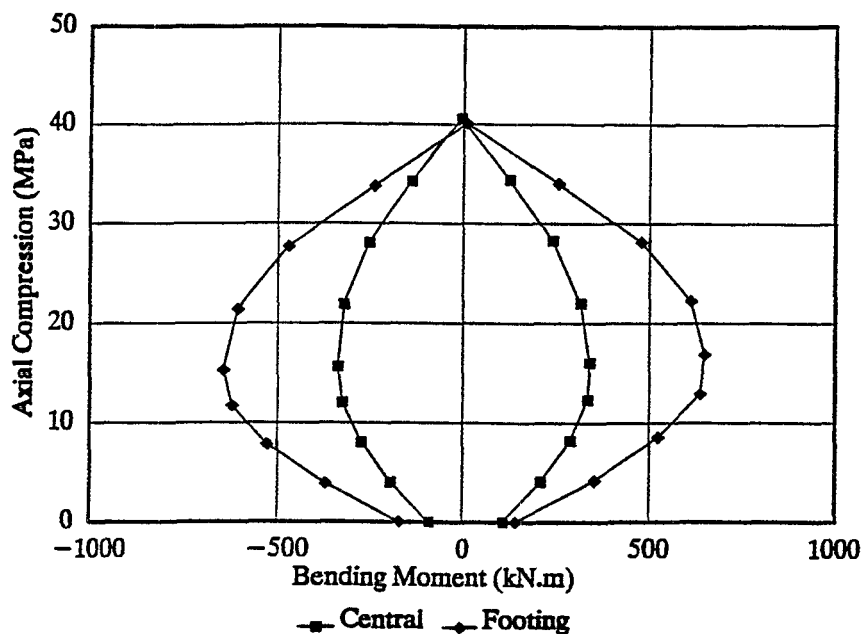


Figure 12. Interaction Diagram for Arch - Circumferential Direction.  
(Positive moments produce tension on bottom face of arch)

## 5.2 SOIL PROPERTIES

Based on a geotechnical investigation, the following soil properties were adopted for the earth cover,

Soil Shear Wave Velocity	=	142 m/s
Soil Compression Wave Velocity	=	350 m/s
Poisson's Ratio	=	0.4
Soil Density	=	1800 kg/m <sup>3</sup>

From these values the elastic and shear moduli were estimated using standard theory. The values adopted in the analysis were,

Soil Elastic Modulus	=	102 MPa	say 100 MPa
Soil Shear Modulus	=	36.4 MPa	

In view of the small deflections experienced by the structure, during the passage of the blast wave, it is considered that the use of elastic properties for the soil is reasonable.

### 5.3 BLAST LOADS

The blast loads applied in the analysis were obtained from a curve fit to the average measured values. The next figure, Figure 13, depicts the variation in peak pressure with distance assumed in the analysis. Averages of the measured values appear on this figure also. The fit to these measured values is clearly good although extensive extrapolation outside this range is involved.

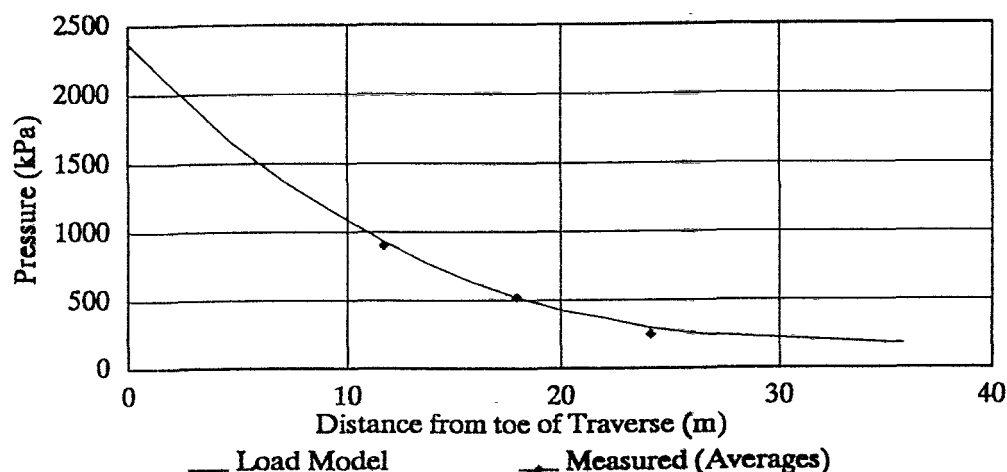


Figure 13. Assumed variation of Peak Pressure compared with measured values.

### 5.4 DYNAMIC ANALYSIS RESULTS

Analysis entailed the direct integration of the equations of motion using 500 time steps of 0.20 ms duration. (Total analysis duration = 100 ms). It should be noted that as the analysis is a linear elastic one - the oscillatory response beyond the first excursion becomes increasingly unreliable and should be treated with caution.

#### 5.4.1 Arch Displacements

A comparison of the analytical and measured response indicates only moderate agreement<sup>5</sup>. Nevertheless the displacement response of the structure summarised hereunder exhibits a number of features consistent with the

<sup>5</sup> In order to permit comparison of measured and predicted response a common time base is required. The analytical predictions are based on an analysis in which the shock front arrival at the crown of the arch matches the average measured value - 52.6 ms.

measured displacement response. Whilst the maximum deflections predicted are comparable to those measured the details of their variation with time is often quite dissimilar. Although a number of the assumptions implicit in the analytical model have been tested for their capacity to influence the results it has not been possible in the time available to achieve an entirely satisfactory result.

Typical plots of displacement of the arch as a function of time for a point on the arch model 84 and the corresponding transducer appear in the next figure. The predicted peak displacements in the vertical and horizontal direction are only 26 mm and 38 mm compared with measured values of 43 and 42 mm respectively. Nevertheless the displacement traces exhibit some consistent features - for example - the effective period of vibration appears comparable.

In both the vertical and horizontal traces the measured response indicates a later but much more abrupt rise in deflection. The predicted vertical displacement exhibits a number of significant harmonics - which are not at all reflected in the transducer result. Although not presented here the predicted displacement plots for a point somewhat lower down on the arch are in reasonable agreement with the measured values for Channels 3 and 4. These discrepancies suggest that the analytical model does not capture the physical behaviour entirely satisfactorily.

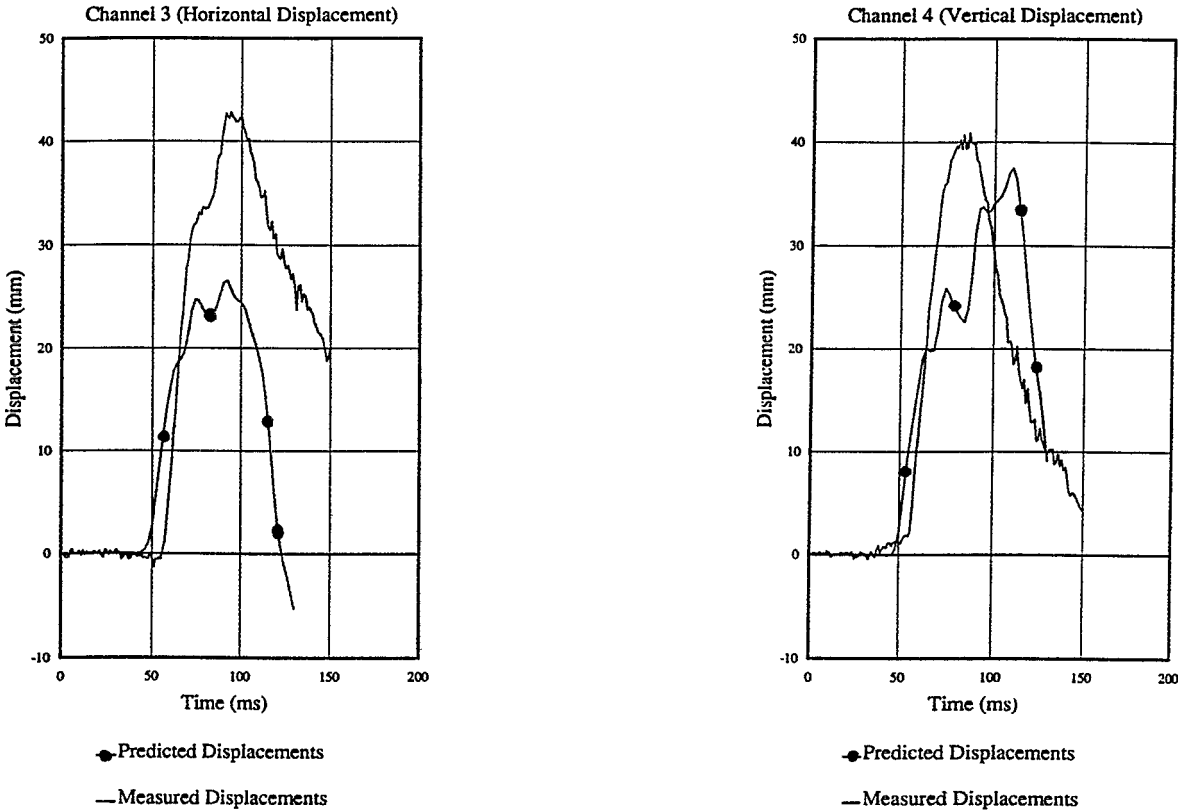


Figure 14. Arch Displacement vs Time - Channel 3 and 4



## 5.4.2 Arch Stresses

Stresses developed within the shell have been plotted on the interaction diagram described earlier above for a typical element within the central circumferential slice. The interaction diagram, Figure 15<sup>6</sup>, indicates that load effects are dominated by flexure rather than axial compression and so some degree of overload is acceptable.

It is also apparent that at times significant tension stresses are developed in the shell and at the footing. This is the consequence of the linear elastic analysis adopted for the solution. In fact as tension loads developed exceed the gravity loads uplift of the footing may be expected with the consequent relief of such loads. Similarly if in the remainder of the arch significant tension stresses were to develop then tension cracking could be expected in the concrete with the consequent relief of such loads. The tension stresses thus depicted in this figure are thus considered to be fictitious.

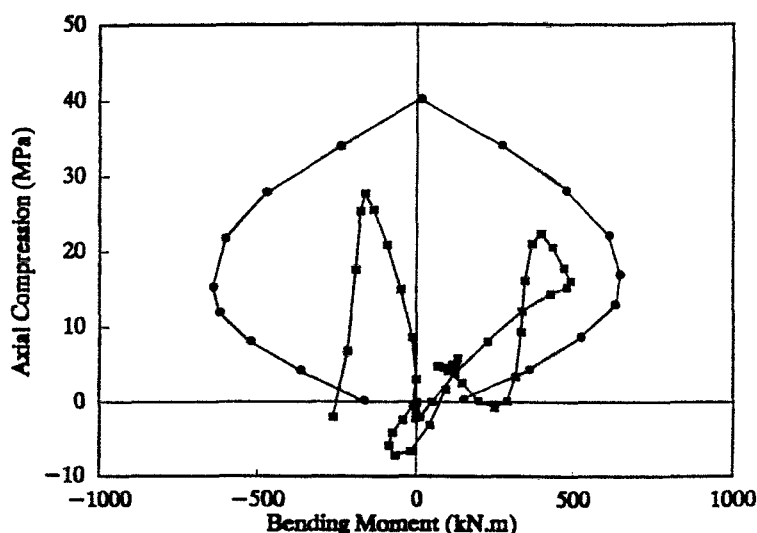


Figure 15. Typical Interaction Diagram Plot for Element

The results obtained for this element, and others not shown here suggest that the real structure, in places, approached and in some areas exceeded the bounds of the interaction diagram. As the overstress generally occurred in areas of relatively low compressive stress, the most likely result is some concrete cracking and yielding of reinforcement. Whilst the results do not suggest any significant risk of collapse they do suggest that at some points significant cracking may have occurred.

In view of the absence of any perceptible permanent deformation in the structure it is considered that the results tend to err on the side of conservatism.

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<sup>6</sup> Note that only blast load induced stresses are plotted in this figure. Gravity induced loads result in an axial stress of the order of 2 MPa, with only minor bending moments, and although these should be included in the load check it is apparent that they will only have a minor effect on the outcome.

## 5.5 COMPARISON OF ANALYTICAL AND INSTRUMENTAL RESULTS

The discrepancy between the analytical and predicted response, whilst not unexpected, is a cause for a re-examination of a number of issues and assumptions.

### (a) Boundary Conditions assumed for the arch

The analysis adopted rotational springs at the arch footing line. The rotational stiffness was calculated using half space theory for the actual footing width and assumed the soil properties determined by test. Previous analyses suggest that any reasonable variation in the end fixity conditions should not affect the results significantly.

Linear elastic boundary elements were also used to provide vertical and horizontal restraint to the footing. Such springs imply a capacity to resist uplift which in practice cannot occur i.e. tension between soil and footing cannot be sustained.

### (b) Parameters assumed for the soil cover to the arch

Although the soil properties were formally determined, the analysis transformed this data into equivalent properties for a linear elastic isotropic medium. Whether a moderately loose soil subjected to severe transient loads can be effectively modelled by a linear elastic model is open to question.

### (c) Anisotropy of the Concrete Section

As noted earlier the SPANTECH sheeting, because of its deep rib, results in a section with quite different properties in the longitudinal and transverse direction. Moreover the stiffness in the transverse direction is dependent on the sense of flexure induced. If bending actions result in transverse tension stresses in the bottom face then only a portion of concrete shell is effective. When bending actions result in compressive stresses - as the ribs are not in ideal contact some reduction in the effective stiffness can be expected.

### (d) Tension in Concrete Shell

The analysis predicts significant tension stresses in the model. Tension stresses beyond say, 4 to 8 MPa, cannot be resisted by the concrete - with the result that some cracking is likely which will thereby reduce the tension stresses and also material stiffness to zero. This effect cannot be modelled by a linear elastic analysis and so the analytical model is likely to be stiffer than the real system.

In summary there are a number of aspects that could lead to a more accurate prediction, however most would require a considerably more detailed model and a program with the capacity to handle the non-linear aspects of the problem. Such analyses would consume substantially greater computer resources than already expended.

## 6. CONCLUSIONS

As a result of this investigation, the following conclusions are drawn,

1. The receptor structure withstood the blast environment generated by the detonation of 75,000 kg in a 13 m span SPANTECH receptor sited at D3 distance with minimal damage and is therefore considered suitable as an hardened ESH.

2. The blast environment generated in the far field suggests that a complete detonation was achieved. However the blast loads measured in the near field are significantly lower than might be expected based on the predictions on the UK ESTC criteria. They are however comparable to the values experienced in the May 90 trial.
3. The better than expected performance of the receptor structures is considered to be partly due to the 'low' blast loads applied and also partly due to the fact that the arch concrete strength significantly exceeded the specified level.
4. Analysis of large span receptor suggests a level of response broadly in agreement with the experimentally determined values. Although it was often difficult to reconcile the predicted variation of displacement with time with the measured response, the predicted peak total displacement of the order of 50 mm was very similar to that determined from the LVDT records. The predicted response for the imposed loads thus confirms that the arch would, at most, experience only minor distress.

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